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SYNOPSIS The paper presents a prediction of liquefaction induced deformation of La Palma dam due to the 1985 Chilean earthquake using a simplified pseudo-dynamic procedure. The procedure is essentially an extension of Newmark's method from a rigid-plastic single degree of freedom to a flexible multi degree of freedom system. It takes into account both the effects of the inertia forces from the earthquake and the softening of the liquefied soil. The results show that the predicted displacements are in general agreement with field observations both in terms of magnitude and pattern of deformations.

INTRODUCTION

On March 3, 1985, at 22:47 GMT, an earthquake of magnitude 7.8 on the Richter scale occurred in Chile. The earthquake was produced by slippage between the Nazca plate and the South American plate that forms a subduction zone at a shallow angle. Many small earth dams within 90 km of the epicenter suffered some damage varying from minor cracks to major deformations. Fortunately, only two of those dams suffered serious damage. La Marquesa dam underwent upstream and downstream slope failures as well as excessive crest settlement leading to a 2 meter freeboard loss. La Palma dam, suffered extensive cracking in the upstream slope causing the upstream crest to settle about 0.8 m. These large deformations were postulated to be due to liquefaction of soils within the dam.

Extensive site investigations were carried out to study the failure mechanisms of La Marquesa and La Palma dams (De Alba et al., 1988). The deformations of these dams were measured and the initial geometry of the dams reconstructed. In addition, based on the final geometry of the dams, the residual strengths of the liquefied sands were back-calculated. The information resulting from the studies make these case histories excellent examples of field performance of earth dam under severe earthquake loading. In this paper, the deformations of La Palma dam are analyzed using the simplified procedure described by Byrne (1991), Byrne et al. (1992, 1994), Salgado and Pillai (1993) and Jitno and Byrne (1994) and the results compared with the field measurements.

EFFECTS OF THE EARTHQUAKE ON LA PALMA DAM

La Palma dam is located about 50 km from Santiago and about 75 km from the epicenter of the earthquake. The dam had an original height of about 10 m, a crest length of 140 m and a crest width of 5 m. The dam was built on the sandy clay and clayey sand foundation soils. The shell of the dam consisted of silty clayey sands obtained from the borrow pits in the reservoir area.

The upstream shell and part of the downstream shell was underlain by a layer of loose silty sand. The core of the dam comprised of more plastic material which extended to the base of the dam. The geometry of the dam before and after the earthquake are shown in Fig. 1.

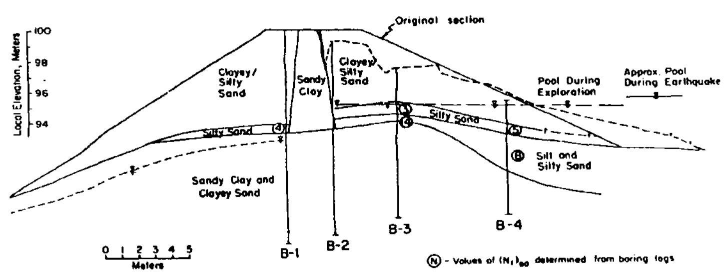

Fig. 1. Pre and post-earthquake geometry of La Palma dam (after De Alba et al., 1988).

Although the location of La Palma dam was quite far from the epicenter, the measured peak ground accelerations at several recording stations near the dam were in the range of 0.23 g to 0.67 g (De Alba et al., 1988). Based on the ground accelerations at these stations and the Chilean attenuation law, the peak ground acceleration at the dam site was estimated to be 0.46 g.

De Alba et al. (1988) postulated that the major cause of the movements was the liquefaction of the saturated silty sand layer which was located immediately below the dam shell. Under an earthquake motion with peak ground acceleration of 0.46 g, the loose silty sand layer liquefied on the upstream side at an early stage of the earthquake shaking. The silty sand layer in the downstream side did not seem to liquefy because this layer was not saturated at that time. The liquefaction of the loose silty sand layer caused a temporary loss of its stiffness and strength. As results, large displacements occurred in the middle third of the upstream slope and the upstream toe moved upstream-ward about 5 m. Moreover, a major longitudinal crack developed along the crest with a maximum width of 1.2 m and length of 80 m. The maximum relative settlement at the crest was about 0.8 m across the crack.

SUBSURFACE SOIL CONDITIONS

De Alba et al. (1988) carried out a site investigation to study the subsurface condition at the dam site. The elevation of the water table at the time of investigation was about 0.5 m below the elevation when the earthquake occurred. The foundation of the dam is clayey sand with normalized standard penetration test (SPT) values, $(N_1)_{60}$, ranging from 5 to 10. Despite low SPT blow counts, this layer is not liquefiable due to its high clay contents. A layer of silt and silty sand with some gravel was found immediately above the foundation soil in the upstream slope of the dam. This layer had $(N_1)_{60}$ values varying from 8 to 14 and apparently did not liquefy during the earthquake. This layer is overlain by a thin layer of loose silty sand with thickness varying between 0.5 to 1.0 m. The average $(N_1)_{60}$ of this layer was about 4 (Fig. 1) with fines contents of 15 percent. The corresponding equivalent clean sand blow counts, $(N_1)_{60-cs}$, was therefore 5. This is the layer that was postulated to liquefy during the earthquake. The same layer of loose silty sand was also found in the downstream slope of the dam. However, this layer was above the water table and apparently did not liquefy during the earthquake.

REVIEW OF CURRENT PROCEDURES

One of the earliest method for predicting earthquake-induced ground deformation is the Newmark method (Newmark, 1965). This method is very simple which is one of the reason why it is so popular among geotechnical engineers. It has been shown to give reasonable predictions for soils that have a potential to develop a distinct failure surface such as dense granular materials (Goodman and Seed, 1966) or rockfill (Elgamal et al., 1990). Elgamal et al. (1990) have shown recently that this method can successfully predict the recorded crest displacements of the La Villita rockfill dam due to several earthquakes. However, the method does not work well for estimating ground displacements where soil liquefaction is involved (Seed, 1979).

With the advance in the art of testing soils under dynamic loading conditions, much more information on the dynamic behavior of soils has been gained in recent years, and has significantly increased the understanding of soil behavior under dynamic loading. Furthermore, the acceptance of the finite element approach for solving geotechnical problems, has led to the development of a more sophisticated methods for predicting earthquake-induced ground deformations. One such method is the dynamic stress path method proposed by Seed and his colleagues (e.g. Serff et al., 1976; Seed, 1979). This method consist of : analyses to determine the static and dynamic shear stresses developed within soil elements in the dam; a comprehensive laboratory testing program to determine the potential strains developed in soil elements under the application of combined static and cyclic loads; further analyses to turn the strain potentials into a compatible deformation field. The method has been claimed to successfully predict the deformation behavior of several dams under earthquake loading conditions (Seed, 1979). However, recent applications of the procedures for seismic stability evaluations of a number of dams (Smart and Von Thun, 1983) reveal that the method sometimes predicts large potential deformations accompanying soil liquefaction which may not develop in the field. It was also noted that the method does not provide any basis for evaluating the residual strength of the soil or the predicted liquefaction zones. Moreover, the method is tedious and expensive which makes it unsuitable for small projects.

A more rigorous method is the coupled dynamic effective stress approach in which both generation and dissipation of excess pore pressure is considered during the prescribed earthquake motion (e.g. Prevost, 1981, Finn et al., 1986; Byrne and McIntyre, 1994). This procedure is even more complex than the dynamic stress path approach as it requires a sophisticated stress-strain model to capture the behavior of the soils. It is state-of-the-art rather than state-of-practice procedure.

There seems a need for practicing engineers to have a simple but reliable method for predicting liquefaction induced deformations of soil structures. The method should be able to capture the essential factors that govern liquefaction-induced ground deformations. The first factor is realistic model of the post-earthquake (post-liquefaction) stress-strain behavior of soils. This is the most influential factor in assessing ground displacements associated with soil liquefaction. The second factor is the inertia effects of the earthquake. Although this factor is not as important as the first one, failure to include it will result in underestimation of the computed ground displacements. The third is the post-earthquake volumetric strains of soils. For cases involving thick liquefied layer in the transition zone between liquefied and non-liquefied grounds, this factor may play a significant role on damage to structures due to differential settlements.

For the above reason, Byrne (1991) developed a simplified approach for estimating liquefaction induced deformations of earth structures. The method is basically an extension of Newmark's method from a rigid plastic to a flexible system. In this paper, the method will be briefly reviewed before it is applied for predicting the liquefaction-induced deformation of La Palma dam.

NEWMARK'S MODEL BASED ON ENERGY CONCEPT

Newmark's method is based on modeling a block with mass M resting on an inclined plane of slope α as a single-degree-of freedom rigid plastic system. His model is shown in Fig.2.a and 2.b. In its simplest form, Newmark considered the earthquake record as comprising of a number of velocity pulses. The pulses cause movements which can be computed from energy principles.

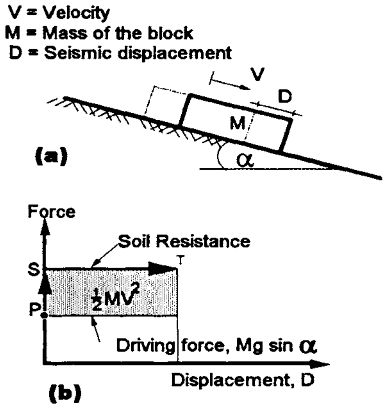

Fig. 2 (a). Block on an inclined plane. (b). Rigid plastic behavior in Newmark model.

Energy principles require that the work done by the external forces (W_{ext}) minus the work done by the stress field (W_{int}) should equal the change in kinetic inertia of the system. This principle can be expressed as :

$$W_{ext} - W_{int} = 1/2\, M(V_f^2 - V^2) = -1/2MV^2 \qquad (1)$$

where V_f is the final resting velocity and is equal to zero, and V is the specified initial velocity.

Byrne et al (1994) have applied this concept to Newmark's sliding block model for a single pulse and shown that the displacement required to obtain the energy balance of the system is given by :

$$D = V^2/(2gN) \qquad (2)$$

where D is the required displacement to obtain the energy balance, g is the gravity acceleration, and N is the yield acceleration of the sliding block.

When 6 velocity pulses are considered, Eq. 2 will be identical to Newmark's formula for an asymmetrical case with $N/A \leq 0.13$ (Newmark, 1965).

EXTENDED NEWMARK

In contrast to the basic assumption in Newmark's method, soil will not behave in rigid plastic manner when triggered to liquefy. The liquefied soil will lose its stiffness when the pore pressure rise causes the effective stress to drop to zero. However, upon straining the soil will dilate causing it to strain harden and regain both stiffness and strength. By incorporating the essentials of this stress-strain response in his model, Byrne (1991) extended Newmark's method to take account for the effects of stiffness reduction in liquefied soils. Idealized pre-cyclic and post-cyclic stress strain curves are shown in Fig.3.

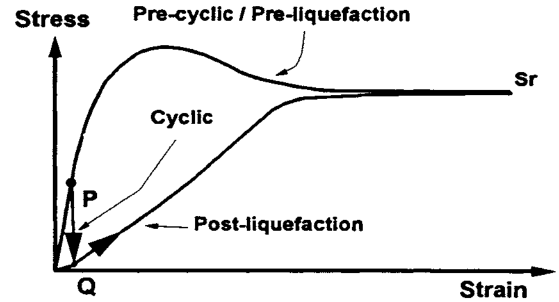

Fig.3. Idealized pre and post-liquefaction characteristics of loose sand.

The strains required to bring a soil element to the zero effective stress state are generally less than 1 percent. Thus triggering of liquefaction is a small strain phenomenon (Byrne, 1991). However, if the soil element is subsequently loaded monotonically, for example due to self weight of non-liquefied soils above it, large deformation may occur due to the very low stiffness at zero effective stress. As the strain increases, the soil dilates causing a drop in pore pressure and an increase in effective stress and stiffness until it reaches the residual state point. If the static stress is larger than the residual strength, flow failure will occur. If the static stress is less than the residual strength, limited deformation (lateral spreading) will occur.

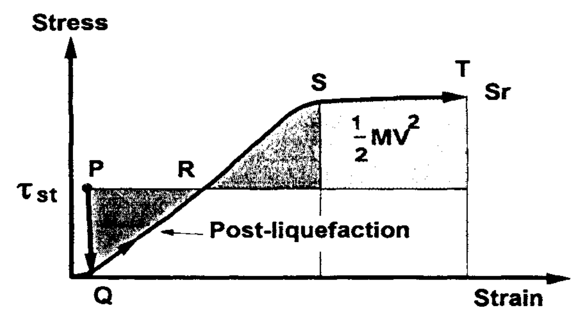

Fig. 4. Work-energy principles, extended Newmark.

The stress-strain characteristic of post-liquefaction loose saturated sand shown in Fig. 3 will now be incorporated into the work-energy approach allowing an extension of Newmark's concept. Point P in Fig. 3 is the pre-earthquake stress state of a soil element in the earth structure. Upon liquefaction, the stress state of the soil drops from its static value P to Q as shown in Fig. 4. This stress change occurs at very low strain as discussed previously. Its resistance then increases with strain to a residual value S_r. The driving force from the ground slope generally remains constant so that the system accelerates as it deforms. Since the system accelerates, it has a velocity when the strain reaches point R where the resistance is equal to the driving stress. Thus, the strain keeps increasing until an energy balance (the external work done by the driving force (τ_{st}) is balanced by the work done by the internal stresses) is reached at point S. If during this process, the system also has a velocity from the earthquake shaking, the soil would deform more until it reaches point T.

Comparing the rigid plastic Newmark approach with the extension to a general stress-strain relation (Fig. 2.b and Fig. 4) it may be seen that the standard Newmark method neglects the displacement from P to S. This could be a very considerable displacement since strains of 20 to 50 percent are commonly required to mobilize the residual strength, S_r. It should be noted that Newmark derived his equation for rigid plastic soils, and it is therefore not applicable without correction to liquefied soils that are very flexible in shear.

For a single degree of freedom system, the displacements can be computed directly by solving Eq. 1 and this is described in detail by Byrne (1991). For a multi-degree-of-freedom system, a pseudo dynamic finite element approach can be used. Detail description of this is given by Byrne et al. (1992, 1994) and Jitno and Byrne (1994).

The procedure has been incorporated into the finite element computer code SOILSTRESS (Byrne and Janzen, 1981) and found to give exact agreement with Newmark when the assumptions made correspond to a single-degree-of-freedom rigid plastic system. It gives good agreement with liquefaction induced field observations reported by Hamada et al. (1987). The procedure predicts the failure of the Lower San Fernando dam and Mochikoshi tailings dams (Jitno, 1994), and gives displacement predictions for the Upper San Fernando and La Marquesa dams that are in good agreement with the measurements in terms of both the magnitude and the pattern of deformations (Byrne et al., 1992; Jitno, 1994). The method was used to predict possible liquefaction induced displacement of the intake structure at the John Hart Dam (Byrne et al., 1991), a tailings dam in Alaska (Byrne et al., 1994) and was also used by BC Hydro to estimate possible liquefaction induced displacements at Duncan Dam (Salgado and Pillai, 1993).

DETERMINATION OF REQUIRED PARAMETERS

The key parameters in this simplified approach are the residual strength of the soil, Sr, the residual strain, γ_{rs}, and the maximum absolute velocity of the mass, V. For thick liquefied layers, post-liquefaction settlement must also be considered. Methods to determine those parameters are described in the following section.

The residual strength of soils can be determined directly by carrying out laboratory tests on the samples from the site or indirectly by correlating the corrected SPT N values for clean sand, $(N_1)_{60\text{-}cs}$, with the residual strength using the relationship presented by Seed and Harder (1990). Similarly, the residual strain can also be determined directly from the results of laboratory tests or by correlation between $(N_1)_{60\text{-}cs}$ and the limiting strain proposed by Seed et al. (1984). For small $(N_1)_{60\text{-}cs}$ (less than 10), however, some engineering judgment is needed to choose appropriate residual strength since the values of residual strength and limiting strains for each $(N_1)_{60\text{-}cs}$ vary quite significantly.

Byrne (1991) proposed an empirical relationship to determine the residual strength of liquefied soils based on the average values of the data presented by Seed and Harder (1990), as follow :

$$Sr = 0.0284\ Pa\ e^{\{0.173\ (N_1)_{60\text{-}cs}\}} \tag{3}$$

with a lower bound value given by,

$$Sr = 0.087\ \sigma_{vo}' \tag{4}$$

for very loose material, a median value given by,

$$Sr = 0.21\ \sigma_{vo}' \tag{5}$$

for $(N_1)_{60}$ = 10 to 12, and an upper bound value given by,

$$Sr = 0.6\ \sigma_{vo}' \tag{6}$$

where ,

Pa = atmospheric pressure,

σ_{vo}' = initial effective vertical stress.

These bounds were obtained from the results of extensive laboratory tests including tests on undisturbed frozen samples of Duncan dam foundation in British Columbia (Byrne et al., 1995).

It should also be noted that Eq. 3 was based on the compilation of residual strength of liquefied soils with $(N_1)_{60\text{-}cs}$ values less than about 15. Thus, extrapolation of this empirical formula for liquefied soils with $(N_1)_{60\text{-}cs}$ value greater than 15 should be viewed with caution.

Similar to the residual strength, Byrne (1991) also proposed an empirical formula for determining the residual strain based on the average values of limiting strains presented by Seed et al.(1984), as follow :

$$\gamma_{rs} = 10^{\{2.2-0.05(N_1)_{60\text{-}cs}\}} \tag{7}$$

The maximum absolute velocity of the soil mass can be evaluated from the relationship between A/V for the earthquakes considered, where A is the earthquake peak ground acceleration in gravity (g) units, and V is the velocity in m/s. Ratio A/V = 1 is generally appropriate for most earthquakes. Thus, maximum velocity of the soil mass can readily be determined if the peak ground acceleration of the earthquake is known. For a deposit comprising thick soil layers, significant earthquake amplification may occur at the ground surface. In this case, it is suggested that a site amplification study be performed to obtain the maximum velocity at the surface.

DEFORMATION ANALYSES

Soil Parameters Used in the Analyses

The soil is treated in the analysis as equivalent isotropic elastic using secant shear (G) and bulk moduli (B) that vary with stress level as follows:

$$G = k_g\ P_a\ (\sigma'_m/P_a)^n\ (1 - \tau\ R_f/\tau_f) \tag{8}$$

$$B = k_b\ P_a\ (\sigma'_m/P_a)^m \tag{9}$$

in which k_g and k_b are shear and bulk modulus numbers, n and m are modulus exponents, τ_f is the failure strength, and R_f is the ratio of the strength at failure to the ultimate strength from the best fit hyperbola, σ'_m is the mean normal stress, P_a is atmospheric pressure, and τ is the mobilized shear stress.

This method requires the pre-and post-earthquake properties of soil properties and these are listed in Table I. The pre-earthquake soil properties were determined based on the $(N_1)_{60}$ values following the approach outlined by Seed et al. (1983) and Byrne et al. (1987). The shear modulus parameters for these soils agree well with those of similar materials published by Duncan et al. (1980). The 10 kPa cohesion of the dam core was estimated from La Marquesa dam which has similar soil properties (Jitno and Byrne, 1994). Due to lack of data, assumptions were made on the values of internal friction angle, ϕ, and unit weight of the soils, γ_s. An estimation of these parameters is usually sufficient since these parameters do not significantly affect the end results.

Table I. Soil Properties Used in the Analyses

Soil Type	kg	n	kb	m	ϕ deg	c kPa	Rf	γ_s kN/m^3
1.	142 (71)	0.5	1000 (1000)	0.25 (0.25)	35	0	0.70	20
2.	142 (71)	0.5 (0.5)	2000 (2000)	0.25 (0.25)	35 (0)	0 (24)	0.80	19
3.	142 (0.15)	0.5 (0.0)	2000 (2000)	0.25 (0.25)	35 (0)	0 (14)	0.70	19
4.	235	0.5	2000	0.25	35	10	0.80	20
5.	235	0.5	2000	0.25	35	0	0.70	18

Note : Number in the brackets indicate the post-earthquake soil properties.

For Case 1, the residual strength of liquefied soil was determined using Byrne's empirical formula (Eq. 3). For liquefied soil beneath the upstream shell, the corrected SPT blow counts, $(N_1)_{60\text{-}cs}$, was about 5 and the computed residual strength was therefore 7 kPa. The residual strain obtained using Eq. 7 for $(N_1)_{60\text{-}cs}$ of 5 was about 89 percent. Therefore, the shear modulus number kg was 0.08.

An alternative residual strength from De Alba et al. (1988) was also considered (Case 2). The range of their residual strength values were between 6 and 14 kPa. The upper bound value of 14 kPa represents the residual strength obtained considering inertia effects. Since the movements took place during the earthquake, it would be reasonable to take the upper bound value of 14 kPa for the residual strength used in the analyses. The corresponding kg value for this residual strength was 0.15.

The shell of the dam was considered to experience 50 percent stiffness reduction due to severe earthquake loading. However, the shell was assumed to retain its original strength after the earthquake. Although the silty sand layer beneath the liquefied layer did not liquefy, it was assumed to develop considerable excess pore pressure due to the shaking resulting in a 50 percent stiffness reduction. The post-earthquake undrained strength of this layer was taken to be 0.6 σ_{vo}'. An average σ_{vo}' = 40 kPa was taken yielding S_r = 24 kPa. The post-earthquake parameters for other soils were assumed to be the same as the pre-earthquake values.

The post-earthquake/liquefaction settlement would be very small for a very thin liquefied layer such as the loose silty sand layer in this dam. Thus, the post-earthquake settlement was not considered in the analyses.

The maximum velocity of the dam was based on the value of the peak ground acceleration of 0.46 g. Using the procedures described previously, this value led to a maximum velocity, V, of 0.46 m/s.

The finite element mesh used in the analyses is shown in Fig. 5. The material types, the approximate water table and the zones of liquefaction are also shown in the Figure.

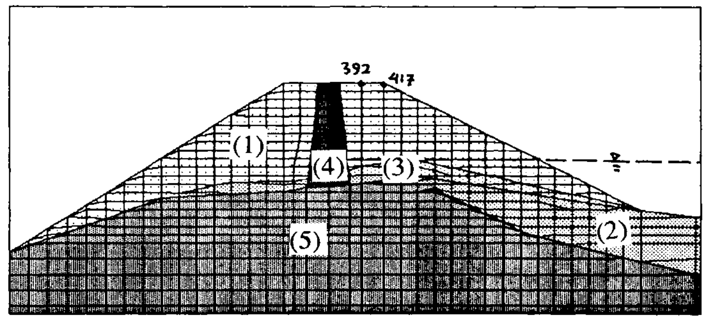

Fig. 5. Finite element mesh, soil types and approximate water table during the earthquake. La Palma Dam.

Results of the Analyses

The predicted dam deformations are presented in Fig. 6 in terms of deformed mesh. Only the result for Case 2 is shown. The magnitude of the displacements for each case are presented in Table II.

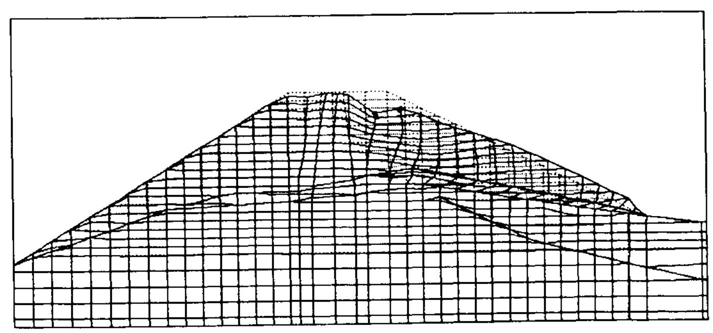

Fig. 6. Results of the analyses. Case 2. Magnification factor = 1.0.

Table II. Measured and observed vertical displacements

Node	Meas. (m)	Predicted(m)		Remarks
		Case 1	Case 2	
392	-1.0	large	-1.4	
417	-1.1	large	-1.1	

As shown in Fig. 6, the procedure correctly predicted the upstream movements of the dam due to liquefaction of the silty sand layer beneath the shell. Most of the deformations developed in the loose liquefied layer. However, large deformations were also observed in the contact elements between the upstream shell and the core resulting in tensile stresses within the soil elements. The tensions in these elements indicate a potential for crack development in this zone which in fact was observed in the field. The downstream shell was predicted to undergo only minor deformations which is in agreement with the field observation.

The use of average residual strength (Case 1) tends to overestimate the computed deformations, as can be seen in Table II. The computed deformations at the crest (node 392) and the shell (node 417) are very large (>4 m). On the other hand, the computed deformations for Case 2 (upper bound Sr) are very close to the measured displacements. The predicted settlement at the crest and the U/S shell are respectively -1.4 m and -1.1 m in comparison to the measured -1.0 and -1.1 m. These results suggest that the upper bound residual strength is more appropriate for this case history.

SUMMARY

A relatively simple pseudo-dynamic procedure has been applied to predict the deformation of La Palma dam during the 1985 Chilean earthquake. The average residual strength from Seed and Harder's chart (incorporated in Byrne's formula) and the upper bound value of residual strength from De Alba et al. (1987) were used. The results show that the procedure overestimates the dam displacements when the average residual strength was used, but closely predicts the field deformations if the upper bound value of residual strength was considered. In addition, the procedure is capable of correctly predicting major movements of the upstream slope of the dam due to liquefaction of silty sand layer beneath the shell.

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